CONCLUSIONS

Based on the information obtained from this investigation, the following opinions regarding structural condition and the proposed construction are rendered:

- The existing power plant structure is not rigidly connected or attached to the mill building. Therefore, the proposed construction of a retaining wall should not disturb the existing structures.
- The mill building's basement wall adjoining the two properties is in poor condition.
- The existing open culverts beneath the mill building foundation wall are hydraulically connected to river flow.

RECOMMENDATIONS

Constructing the proposed retaining wall adjacent to the power plant is considered feasible; however, we recommend the following precautionary measures:

- Due to the poor condition of the existing basement wall adjoining the two properties, the existing wall should remain in place and be properly braced throughout construction of the proposed wall.
- The existing underground brick conduits must be either blocked in place or otherwise re-routed through the proposed wall. Further investigation of the implications of blocking these hydraulic structures is recommended, if blocking is the preferred alternative.

The following options were considered viable approaches for constructing the proposed retaining structure:

1. Soldier pile wall with lagging.
2. Rigid concrete retaining wall.

The first option would require steel H-piles spaced approximately 6 feet on center and socketed into sound bedrock. Additionally, the finished wall would most likely require either tie-backs or struts due to the proposed retained height and apparent depth to bedrock. Tie-backs would extend into the adjacent property and require anchorage into the bedrock, and therefore are not feasible for this project. Struts would require steel supports extending into the river bank and were considered to be costly and unsightly. Therefore, due to costs and aesthetics, we considered this option to be no longer feasible.

We recommend that the proposed retaining wall consist of reinforced concrete stem and foundation supported on micro-piles socketed into the bedrock. We believe micro-piles will provide adequate tensile and compressive strength for the proposed wall foundations and, due to the wall's rigidity, tie-backs or struts will not be required.

CLOSURE

This report has been prepared to assist in the design and construction of an earth retaining wall structure as part of the Village at Little Falls development in, South Windham, Maine. The recommendations have been presented on the basis of an understanding of the project as described herein, and through the application of generally accepted foundation engineering practices. No other warranties, expressed or implied, are made.

We thank you for the opportunity to provide structural engineering services to assist in developing plans for this project. Please call me if you have any questions regarding this report or need any further assistance. We will proceed with developing design plans and details for Option 2 above and according to our agreement unless you provide direction otherwise.

Sincerely,

OAK ENGINEERS, LLC.

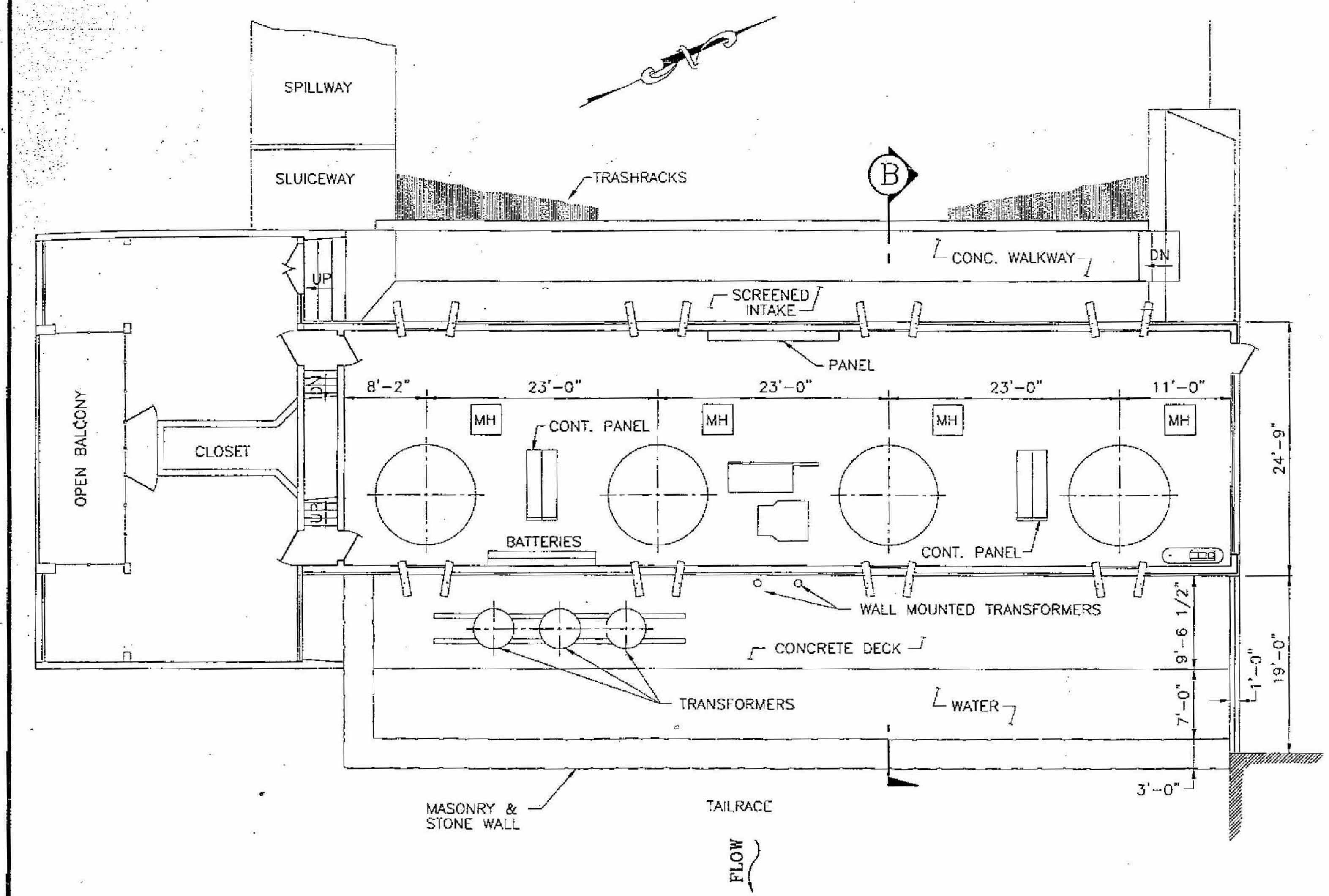
STATE OF MAINE
PAUL
D.
DESTEFANO
9026
LICENSED
PROFESSIONAL ENGINEER

6/12/07

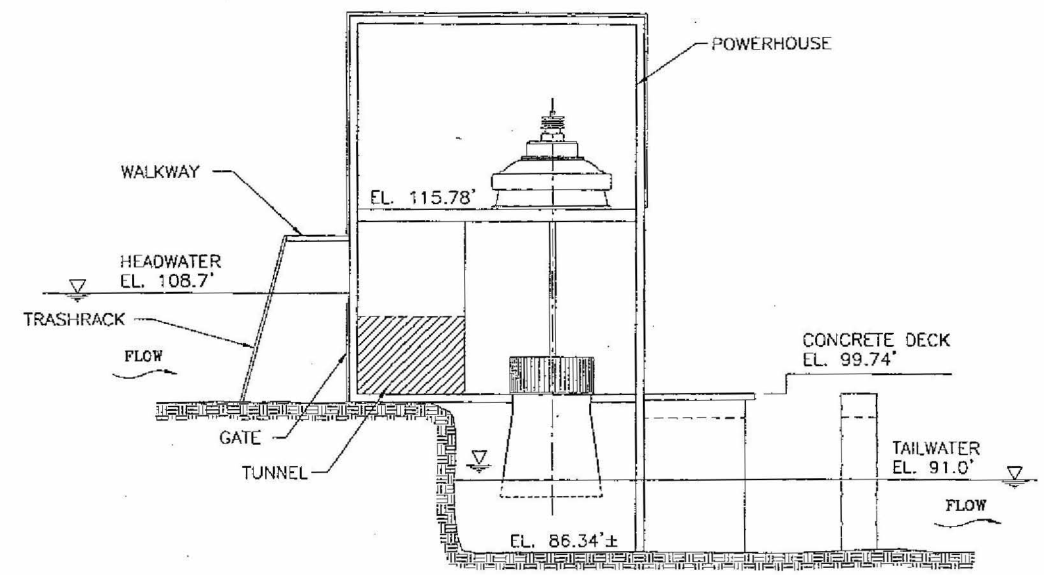
Paul D. DeStefano, Ph.D., P.E.
Director, Geotechnical and Structural Services

PDD:sh
Attachments

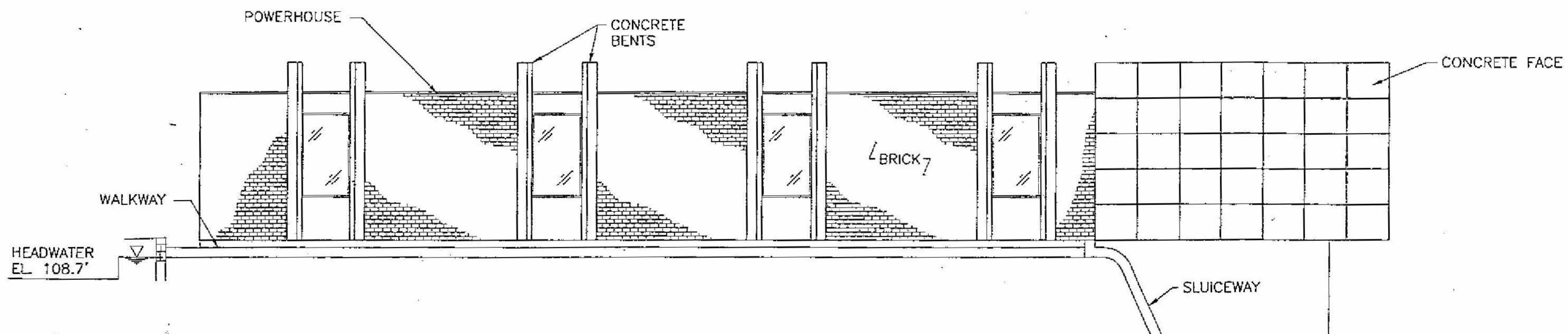
cc: Steve Etzel, Questor, Inc.
S. D. Warren Company



POWERHOUSE PLAN



SECTION B



POWERHOUSE ELEVATION

8 0 8 16
SCALE IN FEET

NO.	REVISIONS	MADE BY / CHK.BY	DATE

Kleinschmidt Associates
Consulting Engineers
Pittsfield, Maine

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S. D. W. EQUIPMENT SYSTEMS NUMBERS

814-01.60-013
876-000.60-004
876-118.60-000
814-01.60-000

HYDRO ELECTRIC
LITTLE FALLS
POWERHOUSE PLAN AND SECTION

S. D. WARREN CO.
WESTBROOK, MAINE

DESIGN		SCALE	AS SHOWN
DRAWN	HWF	JOB ORDER	
APPROVED		DEPT	814-01
CHECKED	MCS	DATE	12-8-97
DWG NO.	CB-63341	SHT NO. 2 of 2	REV 0

EXHIBIT F

SHEET 2 OF 2

LITTLE FALLS PROJECT
FERC NO. 2941

POWERHOUSE PLAN
AND SECTION

S.D. WARREN COMPANY
WESTBROOK, MAINE

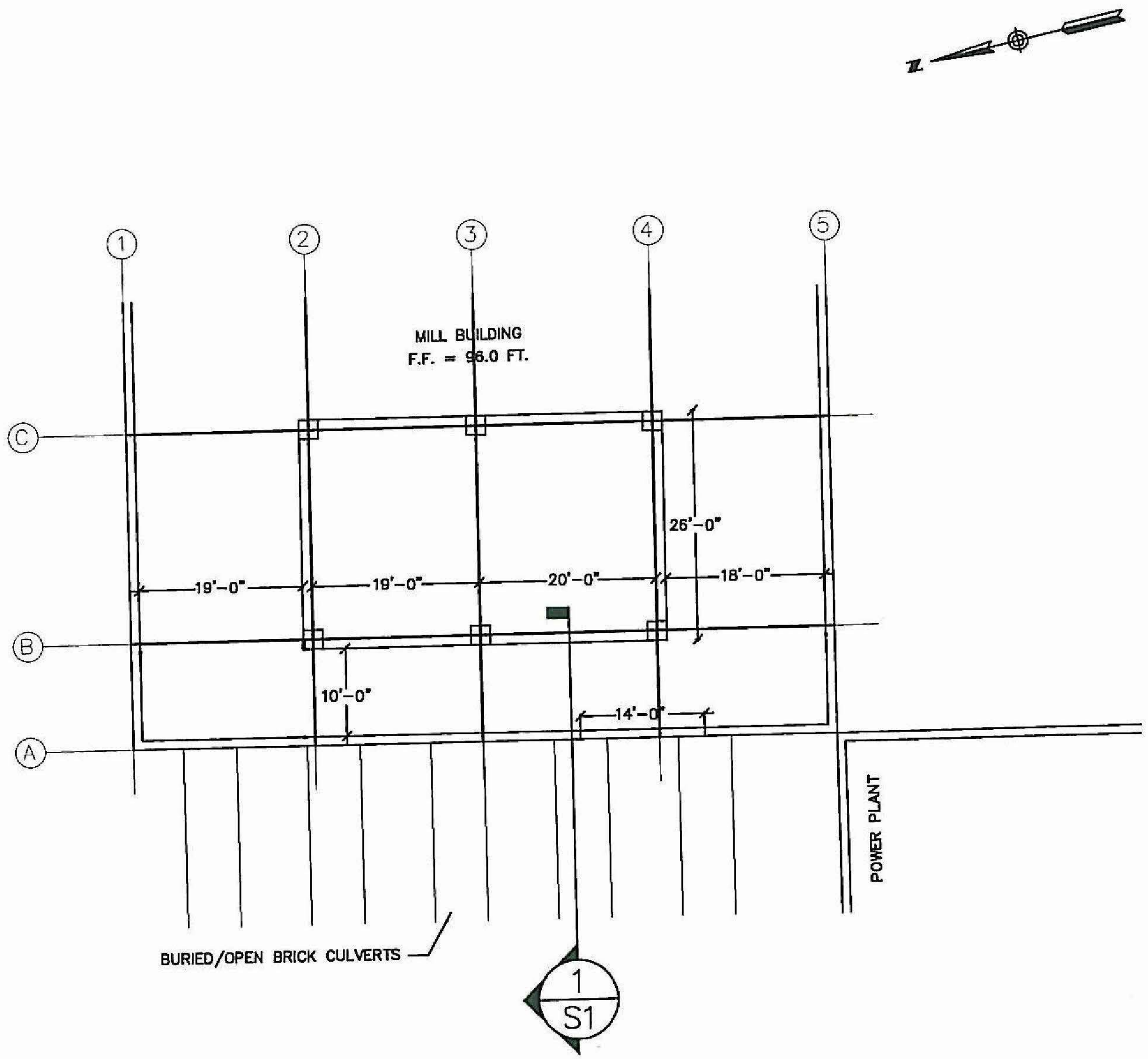
VIL_RESP01755

THIS DRAWING IS A PART OF THE APPLICATION
FOR A LICENSE BY THE UNDERSIGNED ON
THIS 20th DAY OF Jan. 19 99
BY Thomas P Howard S.D. WARREN.

KA 023-057 12/98

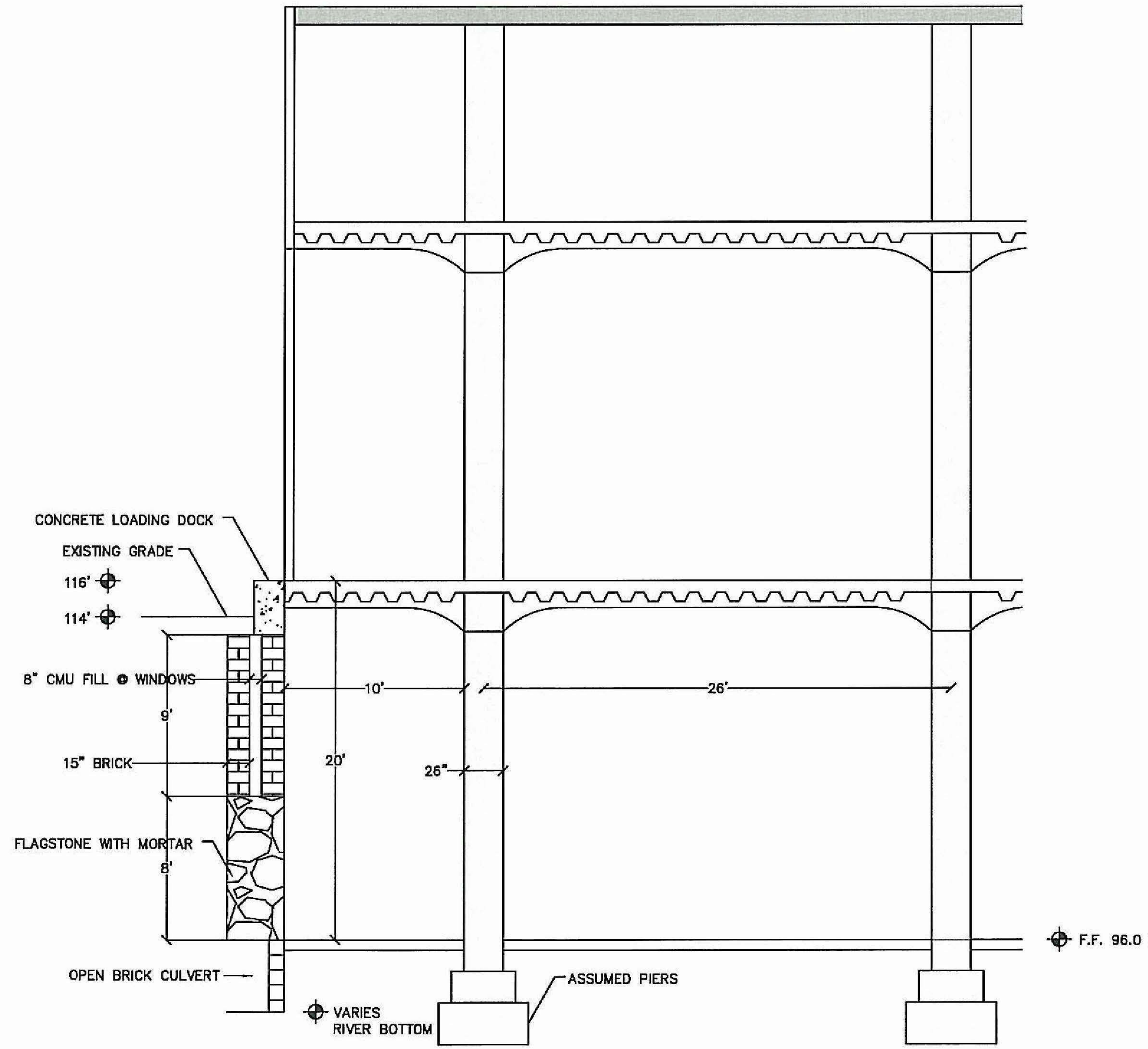
Kleinschmidt Associates Consulting Engineers Pittsfield, Maine	SHEET NO. OF B-	Drawn by: TWG Date: 12-08-97 / Checked by: Date: / Approved by: Date: / Scale: AS SHOWN	Date	Chkd.	Revision

NOTE: ALL ELEVATIONS ARE U.S.G.S. DATUM



PARTIAL SITE PLAN

SCALE: 1" = 40'



TYPICAL SECTION

SCALE: $\frac{1}{8}$" = 1'-0"

1
S1

VIL_RESP01756

OAK ENGINEERS Brown's Wharf Newburyport, MA 01950 (978) 465-9877	PREPARED FOR: NORTHEAST CIVIL SOLUTIONS 153 US ROUTE 1 SCARBOROUGH, ME 04074	SITE: VILLAGE AT LITTLE FALLS 13 DEPOT STREET SOUTH WINDHAM, MAINE	DATE: JUNE 2007 PROJECT: 064006 FIGURE: 1

O:\2006\064006\Drawings\06400600.dwg

February 27, 2007 Project 064006

Lee D. Allen, P.E.
Northeast Civil Solutions
153 U.S. Route 1
Scarborough, Maine 04074

RE: Geotechnical Investigation
Village at Little Falls, LLC
7 to 13 Depot Street
South Windham, Maine

Dear Mr. Allen:

Oak Engineers, LLC (Oak) has completed a geotechnical investigation of the above site in accordance with our agreement entitled *Geotechnical and Structural Engineering Services* authorized on January 3, 2007. The purpose of this investigation is to provide geotechnical design recommendations related to the proposed construction at the above location (the Site).

PROJECT REQUIREMENTS

We understand that the existing Site will be developed into a multi-unit assisted-living community. According to proposed site *Grading and Drainage Plan* by Northeast Civil Solutions (Site Engineer) dated February 16, 2007, the development will consist of twenty-five, one- and two-story, wood-framed residential structures, a three-story apartment building with parking in a below-grade basement level, and associated access roads and driveways as depicted in Figure 2 of Attachment A.

The existing topography consists of rolling terrain and previously developed land. According to the proposed grading plans, a maximum of approximately 20 feet of fill and 15 feet of earth cut will be required to level the site beneath the proposed buildings and pavements. Based on revised planes, we understand that the existing site structures and building will be completely demolished and disposed off site. The Maine Department of Inland Fisheries and Wildlife has required that the proposed development restore the riverbank along the Presumpscot River upon demolition of the existing mill building. In accordance with this requirements, the riverbank area is to be reconstructed to a slope with maximum grades of 2H:1V. The toe of slope will be stabilized with riprap, while the remainder of slope will be stabilized through a series of vegetative techniques recommended by the US Army Corp of Engineers (ACE) when stabilizing riverbanks. Additionally, a permanent earth retaining wall extending as much as 26 feet above adjacent grades will be required adjacent to the existing power plant and river.

According to the site *Grading and Drainage Plan* and conversations with the site engineer's office, the proposed storm water system will be a watertight underground storage system composed of 5-foot diameter pipes located at station 51+00 right, between the proposed homes and the Presumpscot River.

Based on our understanding of the proposed construction, maximum anticipated foundation loads are estimated as follows:

1. Interior Columns = 80,000 pounds

2. Exterior Columns = 60,000 pounds

3. Load Bearing Walls = 2,000 pounds/foot

4. Floor Slabs = 50 pounds per square foot (psf) or 3,000 pound concentrated load

Maximum total and differential building foundation settlement tolerable is assumed to be one inch and one-half inch respectively.

DESCRIPTION OF SITE AND GEOLOGY

The Site is approximately 8.0-acre in area and located on the south side of Depot Street in South Windham, Maine. A Site Location Plan is shown on Figure 1. The Site is currently developed with an abandoned, three-story, concrete and masonry, mill building bordering the north and east banks of a bend in the Presumpscot River. The building is approximately 60,000 square feet in plan area and abuts an existing power plant structure associated with the adjacent Little Falls dam. Three, one-story, wood-framed buildings are also located on the northeast corner of the proposed development.

Existing site grades decrease to the south and east, towards the abutting Presumpscot River. Based on Northeast Civil Solutions (Site Engineer) site plans, grade elevations range by approximately 40 feet across the Site, with the highest elevations of 132 feet (NGVD 29) located near Depot Street on the northeast corner of the property and the lowest site elevations of 92 feet being located along the banks of the Presumpscot River. A Subsurface Exploration Plan depicting the proposed construction along with existing site topography is shown as Plan C1 in Attachment A. Final building and site grades are currently under development.

According to information provided by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) website, soils in the vicinity of the Site are predominantly cut and fill land (approximately 83 percent of site area) and smaller areas of Hollis series soils (9.4 percent) and Scantic series soils (5.2 percent). Hollis series soil consist of shallow, well drained granular soils formed in a thin mantle of till derived mainly from gneiss, schist, and granite. The Scantic series soils consist of very deep, poorly drained soils formed in glaciomarine or glaciolacustrine deposits on coastal lowlands and river valleys.

Based on a review of *Surficial Geology Map of the Gorham Quadrangle, Maine* (Smith et al, 1999), regional surficial soils likely consist of massive to laminated gray and blue-gray silt and silty clay of the Presumpscot Formation. This soil deposit is variable in thickness from less than 1 meter to more than 50 meters. According to *Bedrock Geology of the Portland 1:100,000 Quadrangle, Maine and New Hampshire,* (Berry, Hussey, et al, 1998), bedrock underlying the Site likely consists of flaggy, bluish to purplish-gray, biotite-quartz-plagioclase granofels of the Hutchins Corner schist formation.

Mr. Lee D. Allen, P.E.
Northeast Civil Solutions

SCOPE OF INVESTIGATION

Subsurface Exploration

In general, subsurface exploration methods consisted of field test pit excavations and soil test drilling. Eighteen test borings (B101 through B118) were advanced with 3¼-inch inside diameter (i.d.) hollow-stem steel augers, at the approximate locations indicated on the attached plan included as Attachment A, to a maximum depth of 32 feet below the ground surface (bgs). Soil samples were obtained from each test boring with split-barrel spoon samplers at continuous and nominal 5-foot intervals as directed by Oak's geotechnical engineer. Standard penetration resistance tests were performed and recorded at each sampling interval in accordance with ASTM D 1586 procedures. At soil boring B114, a single undisturbed soil sample was extracted from the underlying soil layers using a thin-walled Shelby tube in according to ASTM D 1587 procedures. Two 5-foot NQ rock core samples were collected from B104 and B105, from approximately 3 feet to 8 feet bgs. Both the soil and rock samples were returned with the field drilling logs to Oak's office for further analysis and review. Final soil boring logs were prepared by an engineer on the basis of our visual classification of soil samples, laboratory test results, and field drilling logs and are included as Attachment B.

Additionally, ten test pits (TP101 to TP107; TP109 to TP111) were excavated at the approximate locations indicated on the attached plan included as Attachment A, to a maximum depth of 6.5 feet bgs. Soil samples were reviewed and classified in the field in accordance with ASTM D 2488 Visual-Manual Procedure. Final test pit logs were prepared by an engineer on the basis of our visual classification of soil samples and field test pit logs and are included as Attachment B.

Laboratory Testing

Soil samples were visually classified by a geotechnical engineer in general accordance with ASTM D 2487 Unified Soil Classification System (USCS) in Oak's office. Selected split spoon and Shelby tube soil samples were transported to certified soil testing firm's offices (John Turner Consulting, Inc., of Dover, New Hampshire and Geotesting Express, of Boxboro, Massachusetts) for laboratory analysis and testing. Laboratory testing included sieve analyses, Atterberg limits, and moisture contents for submitted split spoon samples. Additional testing included consolidated undrained (CU) triaxial compressive strength and consolidation testing from Shelby tube samples. All testing was conducted in accordance with accepted ASTM procedures. Complete laboratory analysis and test results are included in Attachment C.

Geotechnical Evaluation

The geotechnical engineer evaluated subsurface conditions relative to the proposed development on the basis of field reconnaissance and subsurface exploration, project description, local geology, and laboratory analysis and testing in accordance with generally accepted geotechnical engineering principles and practices. According to our agreement, the geotechnical engineer evaluated conditions and provided recommendations for the following project elements:

1. Site Preparation

2. Building Foundations

3. Excavation and Dewatering

4. Earth Retaining Structures

5. Underground Utilities and Subsurface Infiltration Systems

6. Floor Slabs on Grade

7. Pavements

8. Fill and Backfill

9. Construction Quality Control

SUBSURFACE CONDITIONS

Soil Test Boring and Test Pit Results

Apparent Subsurface Profiles of the proposed construction and existing topography and interpreted soil profiles are shown as Plan C2 in Attachment A. A summary of ASTM D 2487 soil classifications for samples recovered from all test borings is shown in the table below. A description of each soil classification is defined in Attachment B.

Table 1: Summary of ASTM D 2487 Soil Classifications

Depth (ft.)		B101	B102	B103	B104	B105	B106	B107	B108	B109	B110
From	To										
0	2	SM	SM	SM-ML	SM	SM	ML	ML	ML	SW	ML
2	4	SM	ML	SM-ML	ML	ML	ML	ML			ML
4	6	CL	ML	SM-ML			ML			ML	ML
6	8	CL	ML	SM-ML							
8	10	CL		GM-SM							
10	12	CL		GM-SM							
15	17	CL									
20	22	CL									

Mr. Lee D. Allen, P.E.
Northeast Civil Solutions

Depth (ft.)		B111	B112	B113	B114	B115	B116	B117	B118	B119
From	To									
0	2	SM	SM	GM-SM	SM	SM	SM	SM	SM	SM
2	4	SM		GM-SM	SM	SM	SM	SM	SM	SM
4	6	SM		GM-SM	SM	SM		SM	SM	SM
6	8			GM-SM	SM	SM		SM	SM	ML
8	10			SM	SM	SM		SM	SM	ML
10	12			SM	SM	SM-OL		SM	SM	
15	17			ML	SM	SM		CL		
20	22				CL	CL				
25	27				CL					
30	32				CL					

Soil test boring results were variable across the Site. For the purposes of this report and the related development, the Site is divided into three general areas of similar subsurface profile. The three general areas are shown on drawing C1 in Attachment A and are generally described as follows:

Area 1: property extending to the south along the eastern bank of the Presumpscot River (River bank silty sand and gravel with variable depth to bedrock).

Soil samples from Area 1 generally consisted of silt and fine sand overlaying shallow bedrock. Borings in this area of the property include B104 to B108 and B110 to B112. Auger refusal on apparent bedrock was encountered on this portion of the Site at depths ranging from 1.2 to 6.0 feet bgs. Rock core specimens were obtained from two borings (B104 and B105) in this area of the property.

Area 2: northeastern corner of the property (upland silt over shallow bedrock)

Soil samples from Area 2 generally consisted of olive silt overlaying shallow bedrock. Borings in this area of the Site include B102 and B109 and auger refusal on apparent bedrock was encountered at depths of 7.3 and 7.5 feet bgs, respectively.

Area 3: the central and western portion of the property (lowlands alluvial plain with deep organics and clay).

Soil samples from Area 3 generally consisted of predominantly fine to coarse sand and fine to coarse gravel with trace to some silt. This granular soil stratum often contained concrete, coal ash, and bricks. In borings B113, B114, and B115, these granular soils overlay organic sands and silts with possible river (fluvial) debris, with areas of buried wood and leaves. This organic layer was observed in soil samples from depths of approximately 9 to 18 feet bgs. Underlying the organic soils in this area of the Site was generally a layer of gray to blue gray silty clay and silt deposits. Auger refusal on apparent bedrock was encountered at depths ranging from 17 to 32 feet bgs.

Rock Core Sampling Results

Two rock core samples were collected in borings B104 and B105 from approximately 3 to 8 feet bgs. The recovered rock core samples were comprised of schist bedrock. The dark gray schist was slightly weathered, but foliated, splitting or cleaving readily. The rock core recovery ratio was near 100 percent for both samples.

A rock quality designation (RQD) was calculated for the retrieved bedrock core specimens. The RQD is used to assess the structural integrity of a rock mass and is defined as the cumulative length of rock core pieces longer than 10 centimeters (cm), divided by the total length of the core run. Based upon the bedrock cores obtained in B104 and B105, the RQD values are 68.3 and 73.3 percent, respectively.

Ground Water

Soil samples were generally moist at all depths. Ground water was neither encountered during drilling nor observed after drilling in any boring in Areas 1 and 2 of the Site. In Area 3 of the Site, groundwater was encountered at depths of 8 to 11 feet bgs in all test boring locations.

Laboratory Test Results

Results of laboratory testing are summarized below, with supporting laboratory results included as Attachment C.

Table 2: Summary of Soils Laboratory Results

	Sample/Depth							
	B101, S4 6–8 ft.	**B102, S3 4–6 ft.**	**B103, S5 8–10 ft.**	**B105, S2 2–4 ft.**	**B113, S2 2–4 ft.**	**B114, S9 25–27 ft.**	**B115, S6 10-12 ft.**	**B117, S2 2-4 ft.**
Gravel (%)	--	--	39.5	--	39.1	--	6.4	32.4
Sand (%)	--	--	40.8	--	54.2	--	54.7	42.1
Silt/Clay (%)	--	--	19.7	--	6.7	--	38.9	25.5
Moisture (%)	27.2	26.2	12.5	24.7	13.3	38.7	52.9	6.1
Organic (%)	--	--	--	--	--	--	5.8	--
Liquid Limit	38	20	--	23	--	22	--	--
Plastic Limit	22	--	--	--	--	20	--	--
USCS	CL	ML	GM-SM	ML	GW-SW	CL	SM	SM

Table 3: Summary of Soils Consolidation and C-U Triaxial Test Results

Depth	Preconsolidation Pressure (P_c)	Compression Index (C_c)	Recompression Index (C_r)	Initial Void Ratio (e_o)	Undrained Shear Strength (S_u)	Coefficient of Consolidation (C_v)
B114, 23-25 ft.	3,600 psf	0.2907	0.0448	0.90	930 psf	6.0×10^{-3} in^2/sec

CONCLUSIONS AND RECOMMENDATIONS

The geotechnical engineer interpreted subsurface conditions with respect to the proposed construction on the basis of field exploration, laboratory analysis, and visual classification of soil samples. Design parameters and construction recommendations are provided below according to an analysis of subsurface conditions disclosed by this investigation and accepted geotechnical engineering principles.

In general, the Site is considered suitable for the proposed construction. In Areas 1 and 2 of the Site, native granular or silt soils and underlying bedrock are expected to provide an adequate bearing stratum for shallow foundations and the assumed design loads However, due to proposed significant grade increases and existing subsurface conditions, Area 3 of the Site is considered unsuitable for foundations bearing on conventional spread footings due to compressibility of the underlying silty clay and organics under the proposed fill and building loads. Significant settlement of the existing underlying organic soils and relatively deep compressible clay soils are anticipated due to the depth and area of fill necessary to achieve final site grades. Although primary consolidation settlements are expected to dissipate within a relatively short period of time after placement of the fill, long-term settlements due to the presence of organics and secondary compression of the deep clays are expected to continue for a long period of time after construction. Due to the relatively deep clay deposits and high embankments, site utilities in Area 3 should not be installed until primary consolidation settlements are significantly dissipated.

<u>Subsurface Conditions</u>

In Areas 1 and 2 of the site, native overburden soils generally consist of fluvial silty sand (SM) and silt (ML) deposits overlying shallow bedrock. The relative density of soil samples ranged from loose to firm (medium-dense). Native overburden soils in these areas are considered of moderate strength and low compressibility. Depths to bedrock varied from 1.2 to 6.0 feet bgs in Area 1 and 7.3 to 7.5 feet bgs in Area 2. Based on our interpretation of the recovered rock core samples, the native bedrock appears to be foliated schist and is moderately weathered, hard, and massive. Based upon the shallow depths of bedrock it is anticipated that bedrock excavation will be required in those portions of the Site.

In Area 3, overburden soils generally consisted of very loose to loose granular fill soils (SM, GM-SM) over a layer of sandy soils containing wood timbers, wood chips, leaves, and organics to depths of 13 to 18 feet bgs. These deposits overlay soft native Presumpscot silty clay deposits to depths of 18 to 33 feet bgs. The organic fill and soft clay soils are considered to be of low to moderate strength and compressibility. Permanent ground water levels are anticipated to be well below the proposed excavation levels for building foundations and utilities on site. However, the proposed retaining wall adjacent to the on-site power plant will require foundations that extend below groundwater and the adjacent river and dewatering will be required for installation of foundations.

For the purposes of seismic design, the soil profile on the property is classified as Site Class B (Areas 1 and 2) or E (Area 3) according to *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02) published by American Society of Civil Engineers (ASCE).

Site Preparation

Site preparation should commence by re-locating underground utilities and demolishing all structures within the footprint of the proposed onsite construction. All existing underground utilities located beneath the proposed foundations should be relocated to outside building perimeters. Underground structures beneath the proposed buildings or pavements should be removed to at least 2 feet below proposed foundation and pavement subgrade levels, and 2 feet below finished grades in landscaped areas. The basement area of the existing building should be filled to subgrade level. The surficial soils should then be stripped of all pavements, topsoil, and organics within the proposed building and pavements.

After clearing and stripping the site, subgrades beneath the proposed buildings, pavements, and fill areas should be proof-rolled with several passes of a 15-ton vibratory roller traveling at slow speeds in each perpendicular direction. All weak and unstable subgrades observed by pumping and weaving during proof-rolling or resulting in depressions greater than one-half of an inch after several passes of the roller should be undercut a minimum of 12 inches and backfilled.

According to the schematic site plans, a relatively large volume of fill will be required to level site grades in beneath the proposed building, roads and parking areas in Area 3 of the property. Up to 20 feet of fill will be required to achieve the proposed site grades for the building and parking lot construction. Site grades throughout the property should be increased with imported Fill material as specified herein. Underground utilities and final pavements in Area 3 of the property should be installed outside the building perimeters only after final site grade elevations are established and settlements have substantially dissipated. Detailed requirements for placement of fill and backfill are provided in the following paragraphs.

In Area 3, primary consolidation of the underlying clay soils are estimated to occur over a period of approximately 3 to 5 months after construction of the fill. In order to accelerate the time to dissipate settlements beneath the fill, we recommend that the site be pre-loaded with additional fill. According to our analysis, a pre-loading program consisting of placement of an additional 5 to 7 feet of fill and installation of prefabricated vertical wick drains will accelerate the time to reach anticipated total settlement of the fill and enable construction of pavements and utilities to continue in normal fashion within approximately 1 to 2 months after placement of the pre-load. In order to achieve uniform settlement over the entire construction area, the additional pre-load fill should be placed over an area 10 feet larger in each direction, where possible, than the proposed final grades and sloped according to the recommendations provided herein.

We estimate a substantial amount of pre-load fill soil will be required in Area 3. However, the pre-load material should be reused in embankment and retaining wall fill areas in other portions of the Site, which will reduce the cost of the pre-loading program. It should be noted that due to the presence of significant deep subsurface organics, pre-loading is recommended for dissipating settlements beneath pavements, embankments, and utilities and does not render spread footings a viable foundation option in this area of the property.

Preloading will require a subgrade settlement monitoring program within the proposed construction area during and after construction of the fill and preload in order to determine the actual rate of settlement and projected time for settlements to dissipate. The program should be conducted under the supervision of a geotechnical engineer licensed in Maine.

Excavation and Dewatering

All excavations should be performed according to OSHA Standards (29 CFR 1926 Subpart P). Temporary un-braced excavations completely within the silty fine sand granular layers (OSHA Type C) should be cut no steeper than one and a half horizontal to one vertical (1.5H:1V or 34°) under dry conditions, to a maximum depth of 12 feet.

In Areas 1 and 2 of the Site, where bedrock may be encountered, the bedrock should be undercut a minimum of 12 inches below proposed retaining wall foundation or pad, pavements, bottom of utility, or building subgrade levels and backfilled with structural fill. Based on this investigation, we believe that bedrock encountered on the site will likely require either pre-drilling and splitting or blasting to loosen the bedrock. If blasting is selected as the preferred means of rock excavation, we recommend that a pre-blast survey of all structures and utilities within at least 100 yards of the blast site be conducted. Peak particle velocity of soils adjacent to critical structures and utilities should be monitored and limited to less than 1 inch per second throughout blasting. Blasting should be conducted by certified/licensed blasting firms with at least 10 years of experience demonstrating rock blasting in residential and commercial zones.

Upon encountering bedrock during excavation for footings, basement slabs, or utilities, the earthwork contractor should expose that portion of the bedrock surface that may require blasting. An independent surveyor should provide an elevation survey of the exposed rock surface and the Contractor, Owner, and Engineers should mutually agree upon the quantity of rock excavation prior to commencing with drilling and blasting operations.

Given the nature of shallow bedrock blasting techniques and the resulting conical blast radii, it is generally not feasible to produce a flat, level blasted subgrade with no quantities of overblasted materials. In order to prevent cost over runs and to provide a Contractor incentive for limiting quantities of overblast, we recommend that a pay limit line be set for each area of rock excavation, below which the Contractor is not entitled to additional compensation. The pay limit line should be fixed at 1.0 foot below proposed design subgrades. The lateral pay limit line should be fixed at 2 feet outside of foundations and utility pipelines.

Excavations adjacent to existing structures or property should be properly shored to prevent shifting and/or settlement of these structures or off-site grades. Underpinning existing foundations is recommended for any excavation that extends below and is within a horizontal distance equal to 1.5 times the cut below adjacent foundation subgrades. Shoring and underpinning, if required, should be designed by a professional engineer licensed in Maine.

Surface runoff should be directed away from excavations to minimize dewatering and to protect subgrades from becoming soft and unstable. Any water entering these excavations should be immediately removed from foundation subgrades using sump and pump techniques. Excavation side slopes should be monitored for potential seepage and maintained accordingly.

Foundations

In Areas 1 and 2 of the Site, the soils at proposed foundation grades are considered to be generally of low compressibility and moderate strength, and therefore conventional shallow spread foundations are recommended for building column support. All foundations exposed to exterior or unheated spaces should be placed a minimum of 4.5 feet below the adjacent finished site grades or slabs to provide for adequate frost protection. All interior foundations surrounded by heated spaces should be placed a minimum of 2 feet below floor slabs to provide for adequate bearing capacity. Exposed foundation subgrades should be densified with several passes of a hand operated vibratory roller or heavy plate compactor. Any weak subgrades observed by pumping and weaving beneath the compactor should be undercut a minimum of 8 inches and backfilled with structural fill. Bedrock encountered within foundation subgrades should be undercut a minimum of 12 inches and backfield with structural fill to final footing grades. Final foundation subgrades should be free of all loose rock, soil, water, frost, or other deleterious materials.

Spread foundations supported on properly prepared subgrades may be proportioned for a maximum allowable net bearing pressure of 4,000 pounds per square foot (psf). They should have a minimum horizontal dimension of 3 feet, even if this results in a bearing pressure less than the maximum allowable. Continuous wall foundations should be at least 2 feet wide and otherwise proportioned for a maximum net allowable bearing pressure of 3,500 psf. Maximum total column foundation settlement is estimated to be 1 inch. Settlements should occur immediately after placement of each load increment. Maximum differential settlement is expected to be less than ½ inch.

In Area 3 of the Site, the underlying organic and silt soils are considered to be generally of low to moderate compressibility and strength. Immediate (short-term) settlements due to the placement of 15 to 20 feet of fill on the site are expected to be 3 to 5 inches. Based on our interpretation of subsurface conditions, additional long-term settlements caused by the fill placement and secondary compression of the underlying soils may result in intolerable settlements beneath shallow building foundations. Therefore, conventional shallow spread foundations are not recommended in Area 3.

Considering the subsurface conditions and feasible foundation alternatives, we believe the proposed buildings in Area 3 of the Site should be supported on deep foundations extending to a firm bearing stratum beneath the organic soils and clay layer. Deep foundations should extend to the underlying sound bedrock, which may range from approximately 15 to 30 feet below proposed foundations. Drilled piers would most likely require permanent casing to maintain stable excavations during installation and are not recommended due to their relatively high associated costs.

Economically feasible deep foundation options considered for this site are driven timber, pre-cast concrete and steel piles. Timber piles are considered to be the most economical for this site given the anticipated foundation loads, depth of suitable bearing stratum, and subsurface conditions. Accordingly, Oak recommends that the buildings in Area 3 be supported on timber piles driven to refusal on sound bedrock. Pre-drilling may be required to penetrate through subsurface obstructions if driving stresses exceed the recommended values.

On the basis of our analysis of subsurface conditions and the proposed construction, the following foundation design recommendations are provided:

1.	Pile Section:	Timber, ASTM D25
2.	Species:	Southern Pine
3.	Preservative Treatment:	AWPA C3
4.	Maximum Driving Stress:	3,000 psi
5.	Maximum Design Capacity:	15 Tons/pile
6.	Maximum Effective Driving Energy:	18 Kip-Ft./blow (Single-acting hammer)
7.	Maximum Vertical Batter	1H:10V
8.	Minimum Pile Spacing	2.5 x pile diameter

Piles should be designed and installed according to *Standard Guidelines for the Design and Installation of Pile Foundations* (ASCE 20-96) published by ASCE. For the purposes of bidding, construction documents should require a base bid pile length equal to 35 feet, and unit prices should be provided to adjust for the final in-place pile length. The final pile tip depth should be determined in the field by using an acceptable driving formula or through dynamic pile load testing methods according to ASTM D 4945 (CASE) corresponding to the above allowable load capacity including a factor of safety equal to 2.0. Protective pile tips should be used to prevent damage due to driving through fill, obstructions, or into bedrock.

Floor Slabs

In Areas 1 and 2 of the Site, floor slabs may be constructed over a Base Course material consisting of crushed gravel conforming Maine Department of Transportation (MaineDOT) Specification Item 703.10 and the gradation requirements as follows:

Sieve Size	**Percent Passing by Weight**
2”	100
1”	95–100
¾”	90–100
No. 4	40–65
No. 10	10–45
No. 200	0–7.0

The Base Course should be at least 6 inches in thickness and compacted to 95 percent of the optimum density as determined by ASTM D 1557. Floor slabs may be designed following procedures recommended by the Portland Cement Association (PCA) or American Concrete Institute (ACI) using